Vacuum preloading consolidation of reclaimed land: a case study

J.Q. Shang, M. Tang, and Z. Miao

Abstract: This case study presents the design, operation, and results of a soil improvement project using the vacuum preloading method on 480 000 m^2 of reclaimed land in Xingang Port, Tianjing, China. The areas treated with vacuum ranged from 5000 to 30 000 m^2. The effects of soil improvement are demonstrated through the average consolidation settlement of 2.0 m and increases in undrained shear strengths by a factor of two to four or more. The study shows that the vacuum method is an effective tool for the consolidation of very soft, highly compressive clayey soils over a large area. The technique is especially feasible in cases where there is a lack of surcharge loading fills, extremely low shear strength, soft ground adjacent to critical slopes, and access to a power supply.

Key words: vacuum preloading consolidation, soil improvement, soft clays, land reclamation, prefabricated vertical drains.

Résumé : L'étude de cas présente la conception, l'opération et les résultats d'un projet d'amélioration des sols au moyen de la méthode de préchargement par le vide sur 480 000 m^2 de terrain réhabilité dans le port de Xingang, Trianjing, Chine. Les surfaces traitées individuellement avec le vide variaient de 5 000 à 30 000 m^2. Les effets de l'amélioration du sol par le vide sont démontrés par le tassement moyen de consolidation de 2,0 m et par les augmentations des résistances non drainées par un facteur de 2 à 4 ou plus. L'étude montre que la méthode de vide est un outil efficace pour la consolidation de sols argileux très mous et fortement compressibles sur une grande surface. La technique est particulièrement applicable dans les cas où il y a absence de remblais de surcharge, une résistance au cisaillement extrêmement faible, des fondations molles adjacentes à des talus critiques, et un accès à une sources d'énergie.

Mots clés : consolidation de préchargement par le vide, amélioration des sols, argiles molles, réhabilitation de terrain, drains verticaux préfabriqués.

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Introduction

This case study presents the design, operation, and results of a soil improvement project using the vacuum method on 480 000 m^2 of reclaimed land at Xingang Port, Tianjing, China.

The principles of vacuum preloading consolidation of soft clayey soils were first introduced by W. Kjellman of the Swedish Geotechnical Institute in the early 1950s (Kjellman 1952). When a vacuum is applied to a soil mass, it generates a negative pore-water pressure. When the total stress remains unchanged, the negative pore pressure results in an increase of the effective stress in the soil which leads to consolidation. A schematic of the vacuum preloading method is shown in Fig. 1. The working platform consists of a sand layer through which vertical drains are placed in the soil. A flexible geomembrane liner covers the area and keys into an anchor trench that seals off by clay revetment, providing a watertight seal. A system of perforated pipes is placed beneath the liner to collect water. Specially prepared vacuum pumps capable of generating vacuum in the soil and pumping water–air are connected to the collection system. It is essential that the site to be treated is totally sealed and isolated from any surrounding permeable soils to avoid the loss of vacuum. Membranes leaks must also be avoided. As pinholes or cracks in the geomembrane are difficult to locate and repair, the geomembrane should be checked carefully before it is placed. To obtain and sustain a high vacuum, it is necessary to cover the membrane with water, which will also prevent aging of the membrane and minimize damage from foot traffic and wildlife. When the required preloading pressure is higher than the capacity of the vacuum pumps, a surcharge fill may be used in conjunction with the vacuum method, as shown in Fig. 1. The fill must be free from stones or sharp objects. If a fill is placed on the membrane liner during the vacuum period it may be necessary to add a leak-detection system under the membrane to help locate leaks.

The vacuum method has the following characteristics: (*i*) a vacuum pressure up to 600 mm Hg (80 kPa) can be achieved in practice using the vacuum equipment available, which is equivalent to a fill 4.5 m in height; (*ii*) the lateral deformation of soil is inward due to the suction generated by the vacuum; instead of the "squeeze-out" of soil encountered in a surcharge preloading process, tensile cracks may develop adjacent to the treated area; and (*iii*) there is no need to control the rate of vacuum application to prevent the bear-

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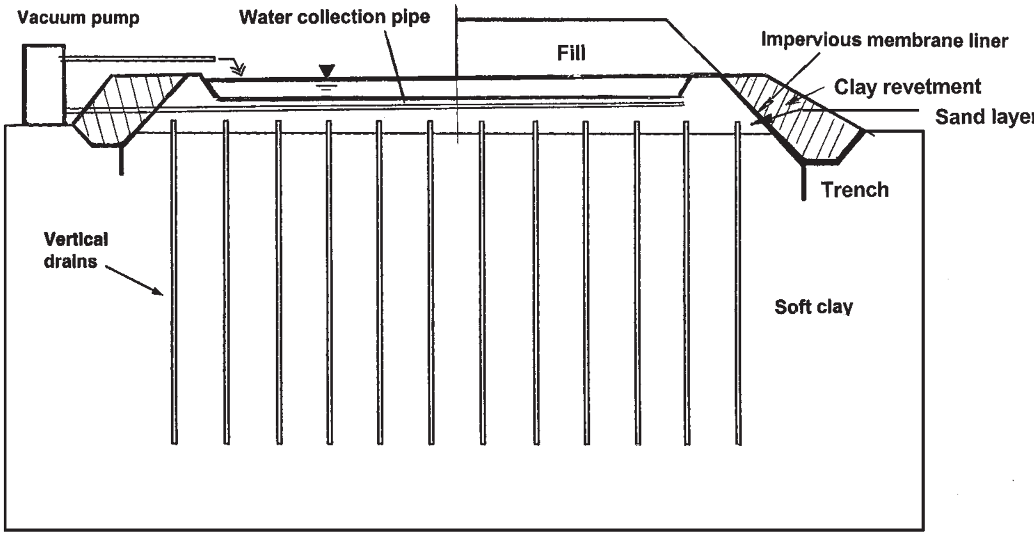
Fig. 1. Schematic of the vacuum method.



ing capacity failure because applying a vacuum pressure leads to an immediate increase of the effective stress in soil.

Despite a relatively good understanding of the principles of the vacuum method (Holtz 1975), the technique was not used widely in geotechnical engineering practice until the early 1980s, mainly because of the cost. The technology gained the attention of the Asian geotechnical community in the early 1980s (Qian et al. 1992) because of advances in geosynthetics and the shortage of land along shorelines. Prefabricated vertical drains (wick drains), which are effective, cost efficient, and easy to install compared to sand drains and packed drains, have made the cost of the vacuum method acceptable even in developing countries. The technique is especially attractive in cases such as highly compressible clayey soils (hydraulic fills, for example), shortage of surcharge fills, soft ground adjacent to critical slope, and large areas with access to a power supply.

Currently, the research on the vacuum preloading method is focusing on aspects such as the numerical modelling of the three-dimensional consolidation process, applications in submerged soils (Harvey 1997), and technical issues such as the implementation and protection of membrane liners over a large treatment area and development of high-efficiency vacuum equipment.

Background

Xingang Port, the major port handling international trade in north China, is located in Tianjing, the third largest city in China. The East Pier, located on the eastern side of Xingang Port, is a trapezoidal-shaped jetty, as shown in Figs. 2*a* and 2*b*. The central part of the pier made of hydraulic fill is 1133 m long and has a total area of approximately 480 000 m^2. The structures to be built on the East Pier included deep-water berths, warehouses, roads, and stacking yards. Since the subsoil consisted of more than 20 m of thick soft clayey soil, including a very soft newly reclaimed surface layer 4 m in thickness, soil improvement was necessary prior to the construction of the structures. The project was funded by a loan from the World Bank via an international tender that involved more than 20 companies worldwide. The 1st Navigational Engineering Bureau of China proposed using the vacuum method assisted with surcharge preloading to improve the soil and was awarded the contract. The project took 29 months from 15 June 1987 to 7 November 1989, 68 days ahead of the scheduled 31 months. The project evaluation by the owner and an international engineering consulting company retained by the World Bank confirmed that all technical specifications in the tender document were fulfilled. No excessive settlement has been observed over the past 6 years since the East Pier was in service.

Geological conditions of the site

Seven boreholes with locations shown in Fig. 2*b* were drilled to determine the geological profile of the subsurface. The soil profile from the west to the east as determined from the borehole logs is shown in Fig. 3, showing the subsoil consists of six layers:

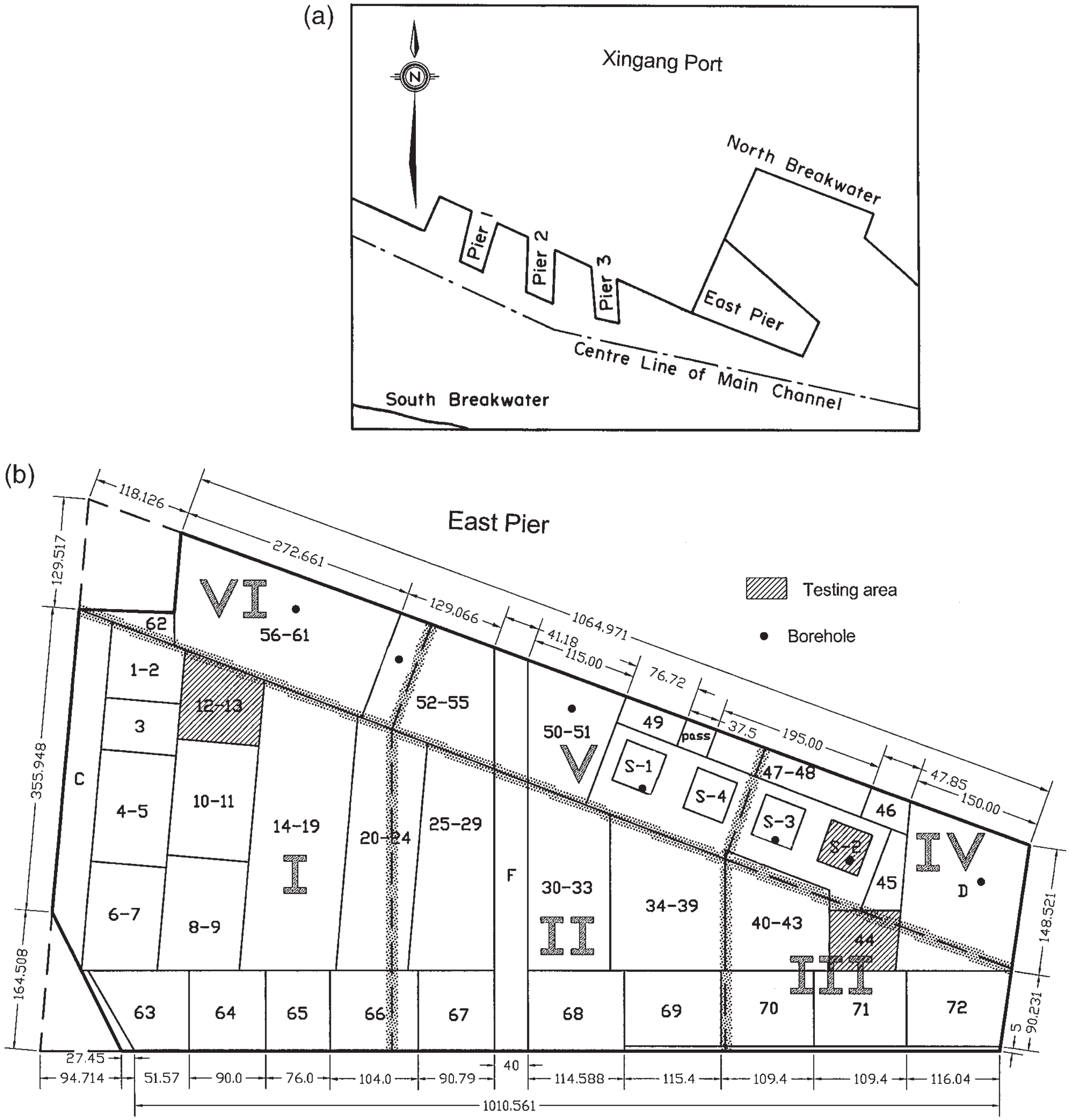
(1) Between elevations +5.7 and +1.5 m: hydraulic fill dredged from the harbour basin and the channel between 1982 and 1986. The clayey fill was still in the process of self-weight consolidation. It was impossible to walk on the surface due to the high compressibility and extremely low shear strength of the fill.

(2) Between elevations +1.5 and –2.0 m: original peat deposit with a local silty clay and clay layer of 1.0–1.5 m in thickness (borehole E (58 and 59)).

(3) Between elevations –2.0 and –6.0 m: soft organic clay intercalated with thin lenses of silty sands. The hydraulic permeability was higher in a horizontal direction. The undrained shear strength of the deposit was lower than 15 kPa. A silty clay layer of 4 m thickness was identified in the central area (borehole E (50 and 51)).

(4) Between elevations –6.0 and –9.0 m: homogeneous organic clay – peat with a water content up to 60%. The hydraulic permeability was nearly isotropic in both the horizontal and vertical directions. The duration of soil improvement was dominated by the low hydraulic permeability of this layer. The undrained shear strength of the soil was approximately 20 kPa.

Fig. 2. Schematic map (*a*) and site layout (*b*) of East Pier, Xingang Port.



(5) Between elevations –9.0 and –14.0 m: homogeneous organic clay with properties similar to layer 4 but with slightly lower clay and water contents. The undrained shear strength of the soil was up to 29 kPa.

(6) Below elevation –14.0 m: silty clay and sandy silt with undrained shear strength higher than 50 kPa.

The index and engineering properties of the soil are summarized in Table 1. The soil water content was higher than the liquid limit in all layers along with very low undrained shear strengths and large void ratios, which are typical characteristics of highly compressive clayey soils. The coefficient of consolidation from conventional oedometer test results was in the range of 0.6 to 1.5 × 10^{-3} cm^2/s, indicating a lengthy period is needed to consolidate the soil. Based on the information from geotechnical investigations, it was decided that vacuum preloading was required for the soil between elevations of +5.7 and –14.0 m.

Design

Divisions and subdivisions

The treatment area was divided into six divisions based on the design loads of the structures to be built; the six divisions are shown in Fig. 2*b* by Roman numerals I–VI. The vacuum pressure was applied on 72 individually sealed subdivisions, marked as 1–72 in Fig. 2. The area of the treatment areas (subdivisions) ranged from 5000 to 30 000 m^2. The past experience has shown that the size of a single

Fig. 3. Soil profile determined for borehole logs. See Fig. 2*b* for borehole locations.

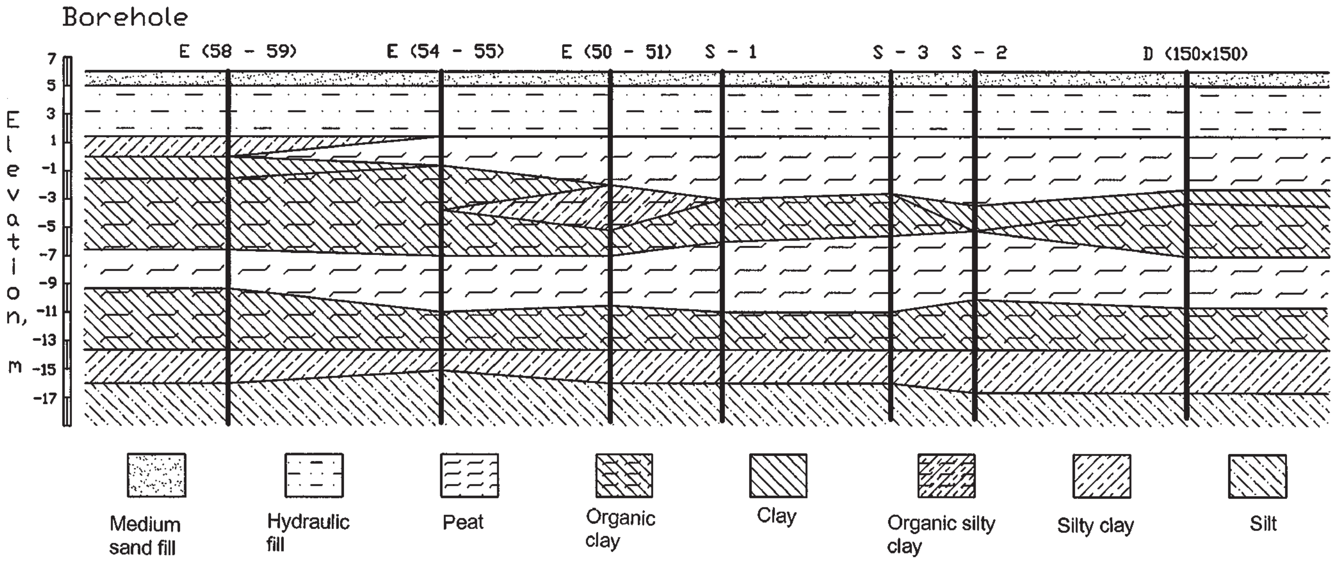


Table 1. Properties of soils subjected to the vacuum preloading treatment.

Soil layer	Elevation (m)	Water content w_n (%)	Unit weight γ (kN/m^3)	Void ratio e	Liquid limit w_L (%)	Plastic limit w_p (%)	Undrained shear strength (kPa): Triaxial (UU) c_{uu}	Undrained shear strength (kPa): Vane c_u	Coefficient of consolidation c_v ($\times 10^{-3}$ cm^2/s)
1	+5.7 to +1.5	63.7	16.8	1.56	40.8	18	3.0	4.5	1.5
2	+1.5 to –2.0	58.0	17.1	1.53	48.1	24.9	6.0	15.0	1.4
3	–2.0 to –6.0	44.0	17.5	1.24	35.0	15	5.0	14.5	2.0
4	–6.0 to –9.0	61.0	16.6	1.65	52.7	27.6	10.3	21.7	0.6
5	–9.0 to –14.0	53.0	17.1	1.45	49.0	25	10.3	29.2	0.8

Table 2. Criteria of soil improvement.

	Division (see Fig. 2*b*)					
	I	II	III	IV	V	VI
Design load p_d (kPa)	50	50	87	83	80	80
Method of treatment	Vacuum	Vacuum	Vacuum + preloading	Vacuum + preloading	Vacuum + preloading	Vacuum + preloading
Preloading pressure p_p (kPa)	80	80	80 + 17	80 + 17	80 + 17	80 + 17
c_u (up to 10 m depth) (kPa)[a]	≥15	≥15	≥20	≥20	≥20	≥20
Residual settlement under design load (cm)	≤20	≤20	≤30	≤15	≤15	≤15

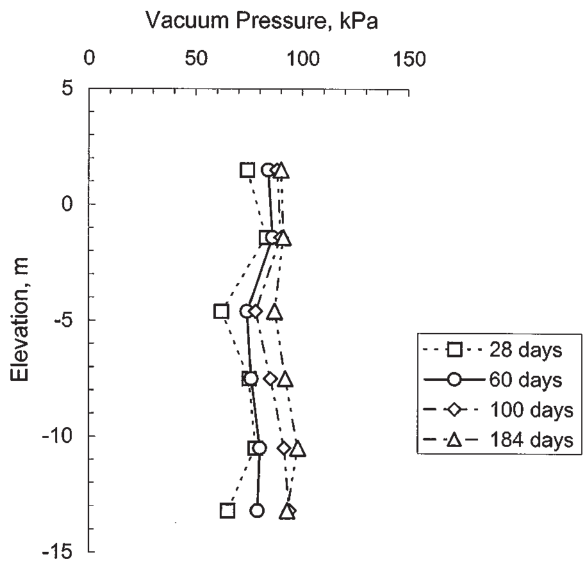
[a] Below 10 m depth, $c_u = c_{uo} + 0.2p_d$, where c_{uo} is the initial undrained shear strength and p_d is the effective overburden pressure.

sealed area depends on the soil condition, capacity of the vacuum pumps, quality of the membrane, and workmanship.

Preloading pressure

The design criteria of soil improvement are summarized in Table 2. The design pressures of the structures to be built ranged between 50 and 87 kPa. The undrained shear strength within the top 10 m of the soil must exceed 15 kPa over the treatment area. The residual settlement under the design pressure due to uncompleted primary consolidation must not be greater than 15–30 cm. The vacuum method was designed for the treatment of divisions I and II, with a target consolidation pressure of 80 kPa, 1.6 times greater than the design pressure. To achieve the design pressure requirement in divisions III–VI, the total preloading pressure of 97 kPa was designed, including a vacuum pressure of 80 kPa and a surcharge of 17 kPa (1 m fill), 1.11–1.21 times greater than the design pressures. To compare with the results of vacuum preloading, four control areas (50 m × 50 m), marked S-1, S-2, S-3, and S-4 in Fig. 2*b*, were designated. Surcharge fill equivalent to 97 kPa was applied in three steps over these areas. During treatment, extensive instrumentation, including surface settlement and vertical deformation with depth, lateral deformation, pore pressure, and vacuum pressure, was implemented in three testing areas, i.e., subdivisions 12, 13, 44, and S-2, as marked in Fig. 2*b*, which were represen-

Fig. 4. Distribution of vacuum pressure with depth at subdivision 44.



tative of the vacuum, vacuum plus surcharge, and surcharge treatments, respectively.

Vertical drains

Prefabricated vertical drains (PVD) made of plastic boards (SPB-2) with a discharge capacity of 25 cm^3/s were selected for the vertical drainage. The embedded depth of the drains ranged between 16 and 20 m. In some areas where the thick soft soil layer was encountered, the embedded depth was extended to 25 m. The spacing of the vertical drains was 1.3 m arranged in a square pattern.

Duration

The application time of preloading pressure, including those of vacuum and surcharge, was designed to be no less than 120 days, provided that 80 kPa vacuum pressure was maintained over the treatment period.

Operation

Pretreatment

The soil surface had to be pretreated because the hydraulic fill was too soft to walk on. The treatment included three steps: (*i*) manual placement of two layers of twig mats; (*ii*) manual placement of 30 cm hill cuts with handcarts; and (*iii*) placement of 40 cm of medium sand. After the pretreatment, the crew and light-weight machinery were able to move into the site to install prefabricated vertical drains.

Installation of prefabricated vertical drains

Mandrel drilling machines were used to install 287 626 PVDs (total length 5 124 851 m) over a period of 18 months.

Installation of drainage pipe system

An interconnected perforated pipe system for water collection was placed on top of the sand layer after installation of the PVDs. The perforated steel or PVC pipes, 76 mm in diameter and 6 m in length, were wrapped with filter cloth and placed in the sand layer. The average spacing between the pipes was 6 m.

Installation of geomembrane liner

Polyethylene membrane liners were keyed into anchor trenches to provide airtight seals. To ensure a proper seal, the anchor trench must reach the organic clay deposit underneath the twig mats, hill cuts, and sands used for the surface pretreatment. The liner was anchored in the trench using a clayey soil which was then compacted manually.

Vacuum preloading

The vacuum pumps used in the project had a capacity of 7.5 MW and were capable of generating a vacuum pressure of 80 kPa over an area of 1000–1500 m^2. After all installations, including membranes, drainage pipes, and vacuum pumps, were completed, the pumps were run for 6 h to check the working condition of the system. The vacuum preloading began in January 1988 and was completed in November 1989. The vacuum pressure was considered to have reached full capacity when 600 mmHg (80 kPa) was registered below the membrane by the pressure meter on the pumping system. Full capacity was reached within 15 days in most subdivisions, with the shortest time being 1 day and the longest time 58 days. Figure 4 shows the distribution of vacuum pressure with depth for one of the PVDs as measured in testing area subdivision 44 (see Fig. 2*b*), indicating the vacuum pressure was uniformly distributed and stabilized after 60 days. Based on past experience, treatment was terminated when the settlement rate was less than 1 mm/day over a period of 10 days. The average treatment time was 135 days in divisions I and II where the vacuum method was used without surcharge, and 175 days in divisions III–VI where the vacuum method was used with the assistance of a 1 m surcharge fill. In some subdivisions, the treatment was extended up to 247 days.

Refill

A layer of refill material (medium sand and hill cut) was placed after the preloading treatment to bring the surface elevation up to +5.4 m.

Post-treatment results

Settlement

The surface settlement for each of the subdivisions was measured using settlement pins installed approximately 3 m from the anchor trenches. The settlement induced by the fill materials (0.3 m hill cuts and 0.4 m medium sand) after PVD installation ranged from 0.6 to 1.2 m during the pretreatment stage. Figure 5 shows a contour map to illustrate the distribution of the consolidation settlement after PVD installation. In general, the surface settlement was fairly uniform, with larger values along the border of the pier and lower values in the central area.

The settlement versus time relationship during the vacuum application period as measured in testing area subdivision 44 is shown in Fig. 6. Settlement under vacuum and surcharge preloading reached 1.4 m after approximately 250 days.

Fig. 5. Contour map showing settlement due to surface pretreatment (0.7 m of fill). All contour lines are in metres.

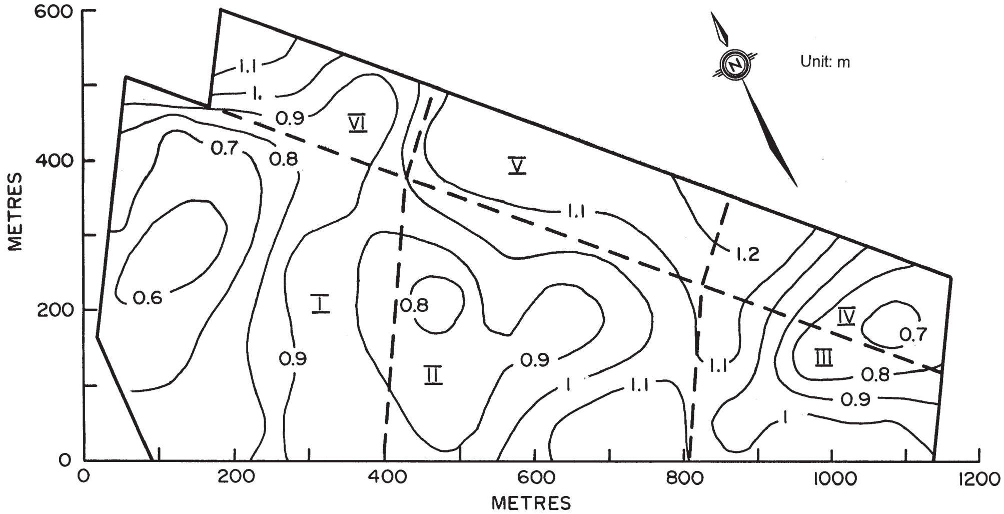
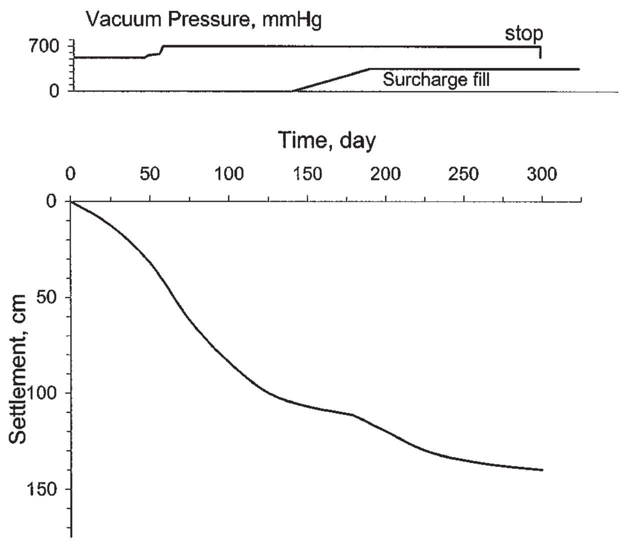


Fig. 6. Settlement versus time in subdivision 44.



The settlement generated during vacuum consolidation periods is presented in a contour map, as shown in Fig. 7. In divisions I and II where the preloading pressure was 80 kPa, obtained by the vacuum method only, the settlements were in the range of 1.0–1.2 m. In other divisions where the preloading pressure was 97 kPa, applied by the vacuum method and 1 m surcharge fill, the settlement was in the range of 1.1–1.4 m. Settlement in four control areas (S-1, S-2, S-3, and S-4) in divisions IV and V did not result in any distinguishable settlement difference compared with other areas subjected to vacuum preloading, indicating the vacuum and surcharge preloading have generated similar consolidation effects.

The final settlement of the soil ranged from 1.6 to 2.3 m and consisted of those induced by pretreatment surcharge and by vacuum preloading treatment, as plotted in Fig. 8. In general, larger settlement was observed in divisions III–VI, which were subjected to the higher preloading pressure (97 kPa), than in divisions I and II, which were subjected to a vacuum pressure of 80 kPa.

Fig. 7. Contour map showing settlement due to vacuum preloading. All contour lines are in metres.

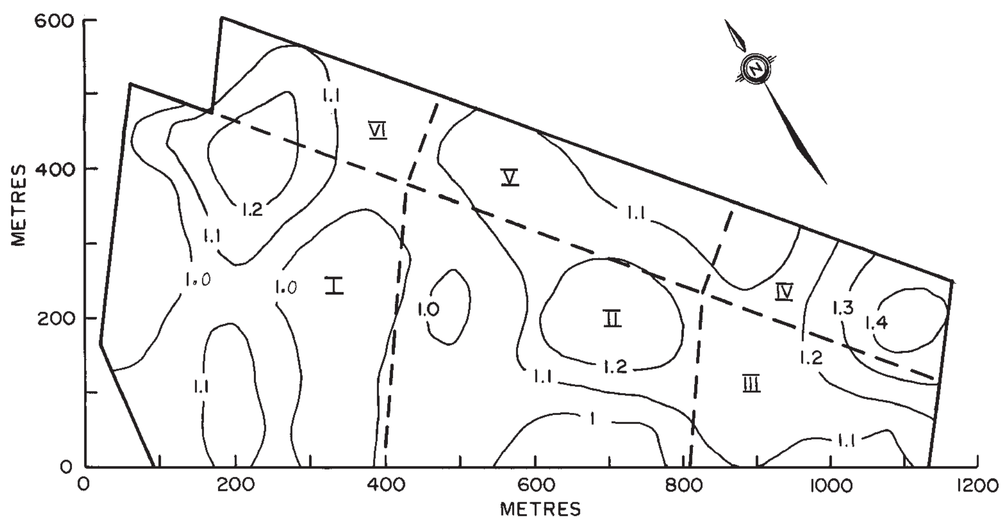


Fig. 8. Contour map showing total settlement due to the combined effects of surface settlement and vacuum preloading. All contour lines are in metres.

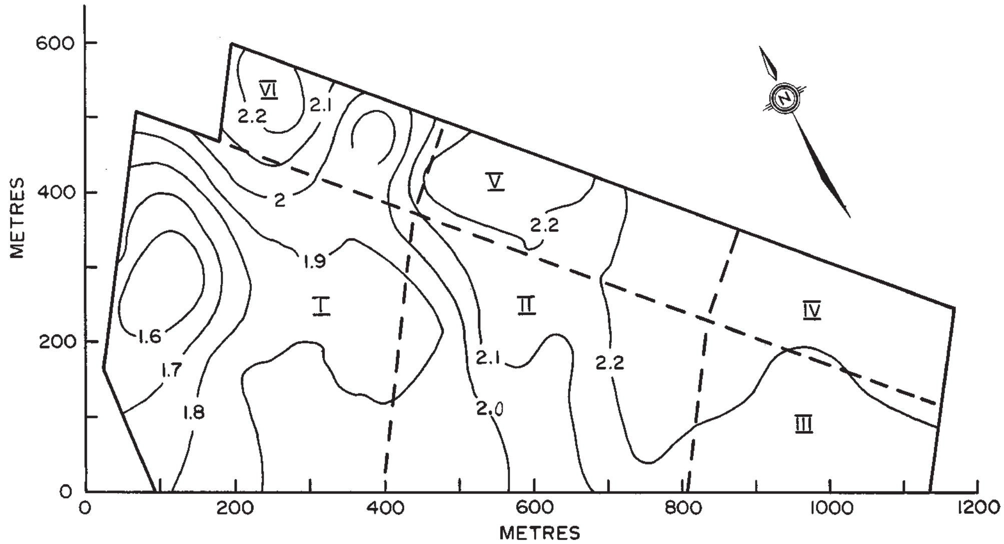
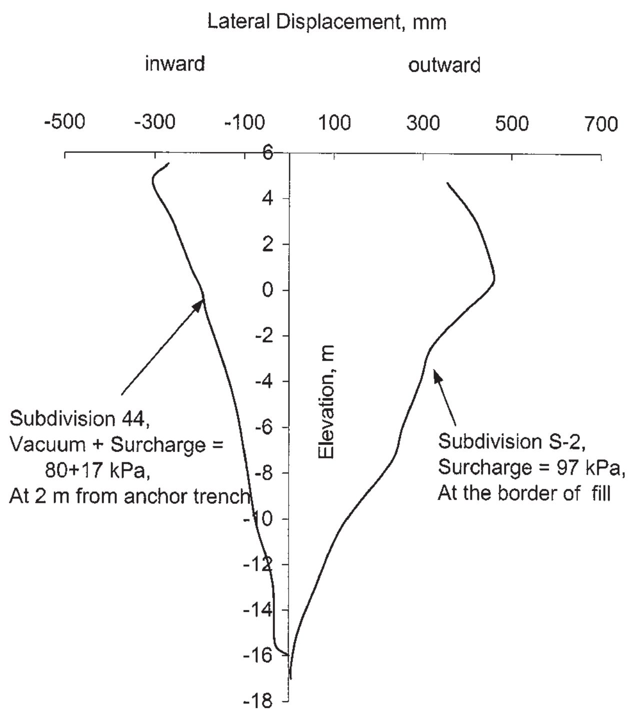


Fig. 9. Lateral displacement distribution with depth in subdivisions 44 (vacuum and surcharge) and S-2 (surcharge only).



Lateral displacement

The lateral displacements of the soil after treatment were measured using inclinometers at testing area subdivision 44 and control area S-2, as shown in Fig. 2*b*. The applied pressure was 97 kPa in both tests; for testing area subdivision 44 this pressure was applied using a vacuum pressure of 80 kPa assisted by a surcharge pressure of 17 kPa, whereas for the control test pressure was applied using the surcharge fill only. Figure 9 shows the distribution of lateral displacement with depth measured at the border of the treatment area. Vacuum preloading generated up to 300 mm of inward lateral displacement toward the treatment area, in contrast with the surcharge preloading which induced an outward lateral displacement of up to 470 mm, i.e., the displacement was away from the treatment area. It is clear, therefore, that the vacuum method does not impose the threat of soil squeeze out and subsequent bearing capacity failure due to rapid surcharge loading. However, it should be noted that the inward lateral displacement could generate tension cracks adjacent to the treatment area. It was suggested that there should be no buildings or other structures within 20 m of the border of the treatment area and lateral displacements should be monitored on nearby structures.

Soil water content and void ratio

The variations of soil water content and void ratio with depth are presented, respectively, in Figs. 10*a* and 10*b* (testing area subdivision 12–13, vacuum), 11*a* and 11*b* (testing area subdivision 44, vacuum plus surcharge), and 12*a* and 12*b* (control area subdivision S-2, surcharge fill). The decreases in soil water content and void ratio after treatment are clearly shown. The average decreases in soil water content over the treatment depth were 17.3, 16.3, and 22.5% in subdivisions 12, 13, 44, and S-2, respectively, with fairly large standard deviations (up to 17%) attributable to the complexity of the soil conditions. The decreases in void ratio were 16.2, 14.9, and 19.9% in subdivisions 12–13, 44, and S-2, respectively, consistent with the decreases in soil water content. The decreases in water content and void ratio became insignificant below elevation –8 m in subdivisions 12 and 13 (Fig. 10). On the other hand, in subdivisions 44 and S-2 (Figs. 11 and 12), the effective treatment depth was extended to elevation –10 m.

Soil shear strength

The soil strength after treatment was evaluated via undrained triaxial tests (UU tests), field vane tests, and the

Fig. 10. Results of soil testing after vacuum preloading in testing area subdivisions 12 and 13. c_u and c_{uu}, vane and triaxial shear strength, respectively.

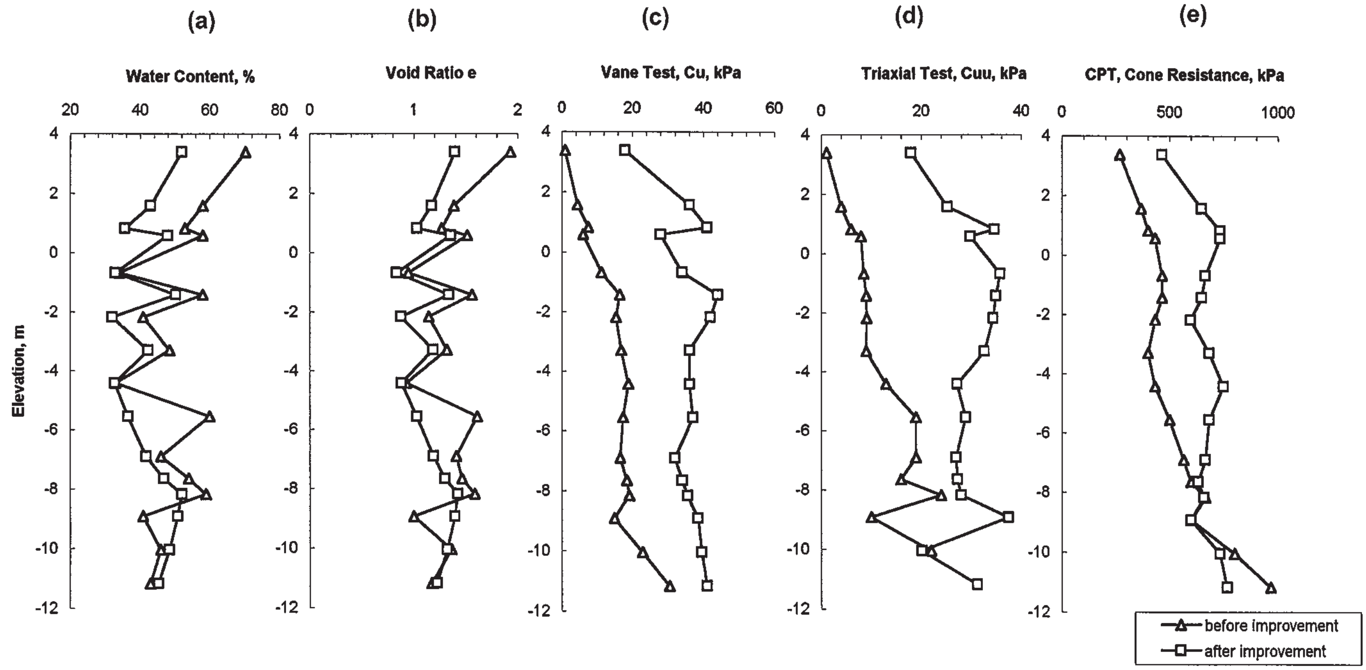
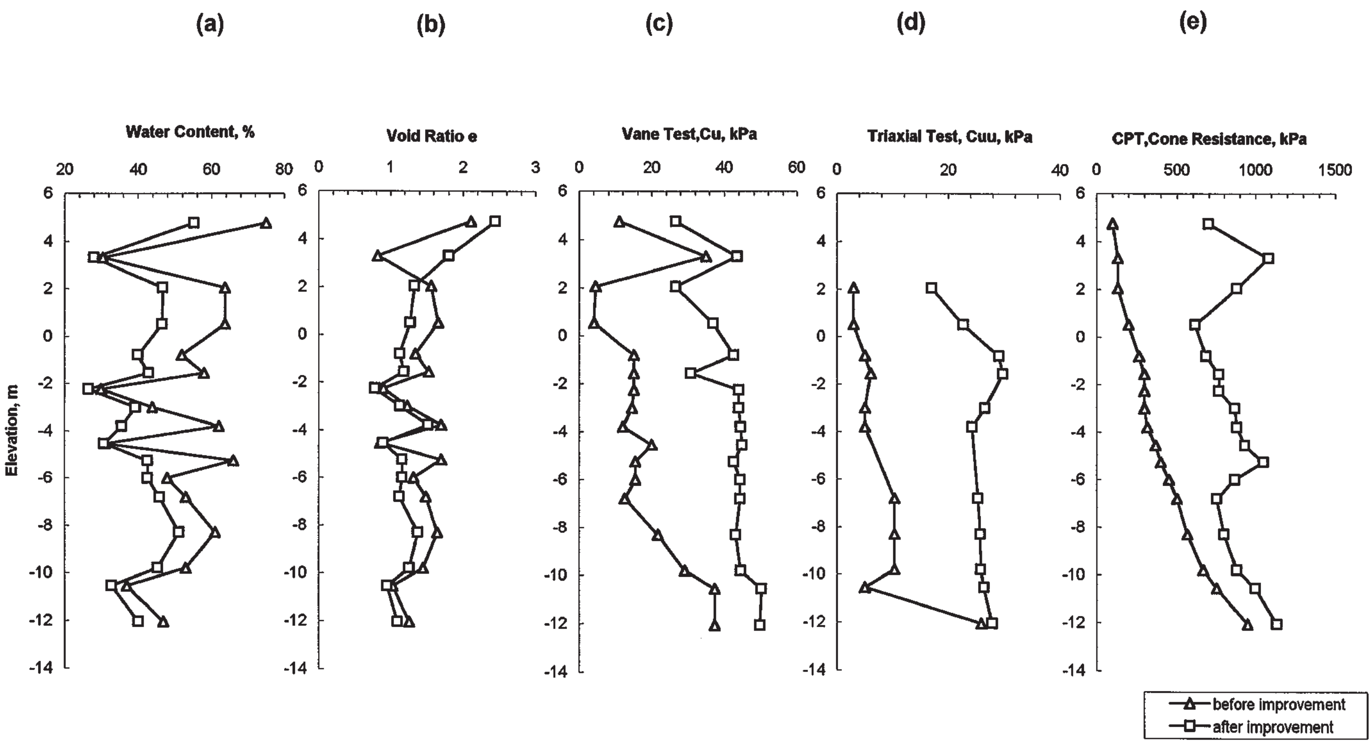


Fig. 11. Results of soil testing after vacuum and surcharge preloading in testing area subdivision 44.



cone penetration test (CPT), as shown in Figs. 10*c*–10*e*, 11*c*–11*e*, and 12*c*–12*e*.

In subdivisions 12 and 13, the undrained shear strengths obtained from the UU tests and field vane tests are in good agreement, showing increases ranging from 1700% at the top to ~30–40% at the bottom of the treatment zone. The design criteria listed in Table 2 are fulfilled over the entire treatment depth. The results of th CPT, as shown in Fig. 10*e*, indicated significant increases in the cone resistance at elevation –12 m and above, whereas no improvement was registered below elevation –13 m, which seems to be in agreement with the water-content and void-ratio data.

Fig. 12. Results of soil testing after surcharge preloading in control area subdivision S-2.

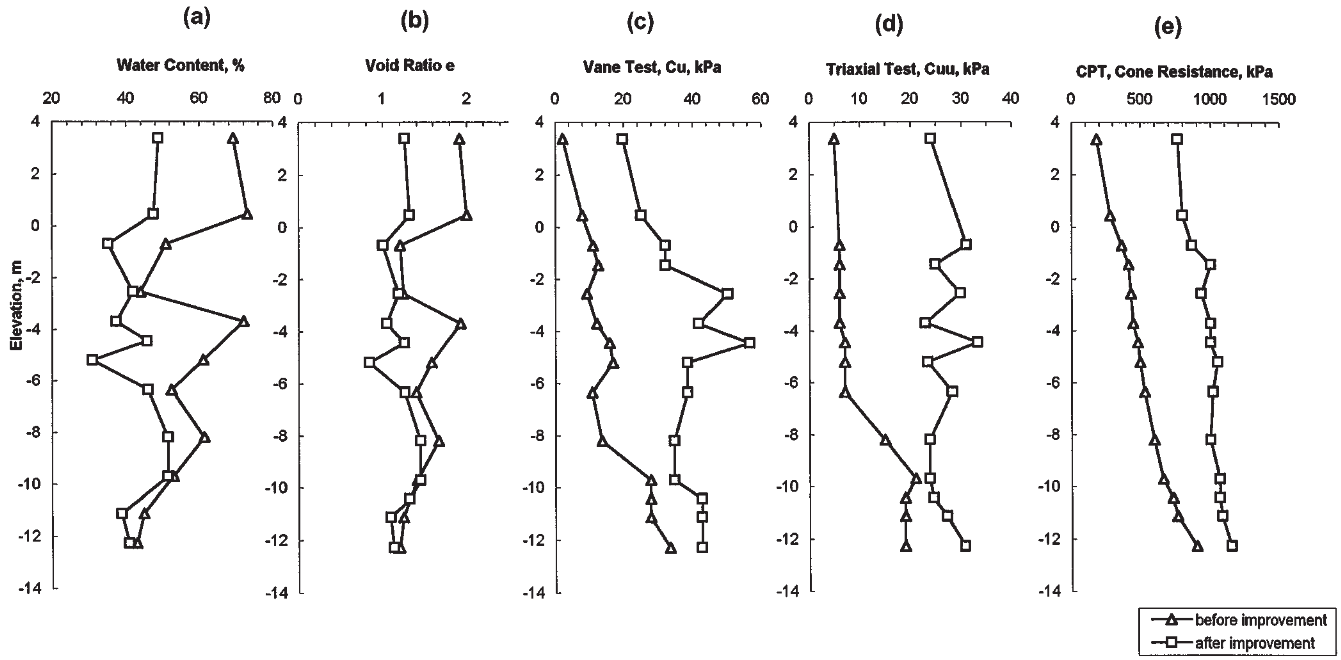


Table 3. Power consumption analysis.

Division	Method	Area covered by one vacuum pump (m^2)	Capacity of vacuum pump (kW)	Average treatment duration (days)	Power consumption per unit area ($kW \cdot h/m^2$)
I, II	Vacuum	1000	7.5	135	24.3
III–VI	Vacuum + surcharge	1000	7.5	175	31.5

In subdivision 44, good agreement was also found between the results of UU tests and vane tests. The vane strength of the soil after treatment ranged from 27 to 51 kPa, compared with the initial values which ranged from 4 to 37 kPa. The design criteria listed in Table 2 were fully achieved. The percent increase in shear strength ranged from 33% at the bottom (elevation –13 m) to 2327% at the surface (elevation +5.5 m). Significant shear strength increases were observed over the entire treatment depth. The results of the CPT tests were consistent with the decreases in water content and void ratio and increases in the undrained shear strengths, as shown in Fig. 11*e*.

The control test on subdivision S-2 generated results nearly identical to those obtained from the vacuum plus surcharge treatment on subdivision 44 in terms of all soil strength indicators, as shown in Figs. 12*c*–12*e*. Therefore, it is confirmed that the design preloading pressure of 97 kPa was realized through the combined application of vacuum and 1 m surcharge fill.

Power consumption

A preliminary analysis of the power consumption is presented in Table 3, based on the fact that the average coverage area of a single vacuum pump with a capacity of 7.5 MW was 1000 m^2. Using the reported average treatment duration, the power consumption per square metre of land was estimated as 24.3 $MW \cdot h/m^2$ in divisions I and II and 31.5 $MW \cdot h/m^2$ in divisions III–VI.

Conclusions

The design, operation, and results of a soil improvement project using the vacuum preloading method on 480 000 m^2 of reclaimed land in Tianjin Port, Tianjing, China, are reported. The individual sealed areas subjected to vacuum treatment ranged from 5000 to 30 000 m^2. The effects of soil improvement were demonstrated through the average consolidation settlement of 2.0 m and two- to four-fold increases in undrained shear strength and CPT resistance. The study shows that the vacuum method is an effective tool for the consolidation of very soft, highly compressive clayey soils over a large area. The technique is especially feasible where there is a lack of surcharge loading fills, extremely low shear strength, soft ground adjacent to critical slopes, and easy access to a power supply.

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